HIGH STRENGTH CONCRETE COLUMNS CAPACITY THROUGH LOWER STRENGTH CONCRETE FLOORS

By

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This is to certify that the

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DEDICATED

TO

MY PARENTS AND TEACHERS

FOR THEIR

SUPPORT, ENCOURAGEMENT,

PATIENCE AND PRAYERS

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ABSTRACT

Nine sandwich column specimens were tested to study difference in behavior of columns in presence of lower strength slab concrete in column-slab joint region. Column portions were made of higher strength concrete as compared to floor concrete. These specimens were divided in three groups, with three specimens in each group. Specimens were also designed to study the influence of aspect ratio on the behavior of column specimens. The data from these tests combined with that from the previously reported similar studies was analyzed to find the appropriate parameters for the estimation of apparent strength of the concrete to be used in the calculation of load carrying capacity of columns.

Mechanics of material approach was used to evolve suitable expressions for estimating apparent concrete strength of corner, edge and interior columns. These empirical expressions can be expressed as:

$$f'_{ce} = \frac{f'_{cc}\, f'_{cs}}{\frac{h}{b}(f'_{cc} - f'_{cs}) + f'_{cs}} \qquad \text{(for corner columns)}$$

$$f'_{ce} = \frac{f'_{cc}\, f'_{cs}}{0.7\frac{h}{b}(f'_{cc} - f'_{cs}) + f'_{cs}} \qquad \text{(for edge columns)}$$

$$f'_{ce} = \frac{f'_{cc}\, f'_{cs}}{0.35\frac{h}{b}(f'_{cc} - f'_{cs}) + f'_{cs}} \qquad \text{(for interior columns)}$$

Where; $h/b \leq 1$

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NOTATIONS

A_g	=	gross area of column cross section
A_{st}	=	area of deformed bar
E_{cc}	=	modulus of elasticity of column concrete
E_{cs}	=	modulus of elasticity of slab concrete
E_y	=	effective modulus of elasticity in Y-axis direction
b	=	least column dimension
f'_{cc}	=	compressive strength of column concrete
f'_{ce}	=	effective compressive strength of column
f'_{cp}	=	apparent concrete strength of column
f'_{cs}	=	compressive strength of slab concrete
f_{sy}	=	yield strength of deformed bar
h	=	thickness of slab-column joint
L	=	length of specimen
L_f	=	length of fiber
L_m	=	length of matrix
p_t	=	axial test load applied to column
δ_{cy}	=	axial displacement of composite in Y-direction
δ_{fy}	=	axial displacement of fiber in Y-direction
δ_{my}	=	axial displacement of matrix in Y-direction
σ_y	=	transverse normal stress
ε_{cy}	=	strain in composite in Y-direction
ε_{fy}	=	strain in fiber in Y-direction
ε_{my}	=	strain in matrix in Y-direction

Chapter 1

INTRODUCTION

1.1 INTRODUCTION

Concrete strength used in construction has been increasing over the years. Strengths of up to 140 MPa (20 ksi) and more have been used in the industry, especially in columns of high-rise buildings. High-strength concrete (HSC) offers advantages in terms of maximum floor space, performance and economy. The use of HSC column sections along the height, with higher-strength concrete placed in lower stories results in additional savings associated with repetitive use of formwork (ACI-ASCE Committee 441R-1997).

The last few decades have seen notable growth in the development and use of high strength concrete. In frame structures, high floor concrete strength is not required thus variation in column and floor concrete strengths is substantial. The common construction practice in high rise building is to cast columns with high strength concrete and slabs with lower strength concrete. The differential strengths of two concretes require due consideration in selecting appropriate concrete strength value in calculations for estimating column capacities. ACI addresses this issue in its Section 10.15 where it recommends no special measures as long as the ratio of column to floor concrete strength (f'_{cc} / f'_{cs}) is limited to 1.4. The present Code provisions consider effective joint strength a function of two concrete strengths but provide no guidelines concerning the effect of other parameters like, specimen geometry, floor thickness, etc.

This research work is carried out to study the effect of floor thickness on transmission of axial loads through nine (09) sandwich column test specimens. The floor concrete portion of the sandwich column was made of ordinary strength concrete, top and bottom column portions from comparatively higher strength concrete. All the other parameters except floor thickness i.e. reinforcement ratio (ρ), specimens' dimensions etc. were held constant.

1.2 TRANSMISSION OF COLUMN LOADS THROUGH FLOOR CONCRETE

The maximum concentric load carrying capacity of the column can be obtained by adding the contribution of the concrete, calculated by $(A_g - A_{st})0.85 f_c^{'}$, and the contribution of the steel which is $A_{st} f_y$. The value of $0.85 f_c^{'}$ instead of $f_c^{'}$ is used in the calculation; ACI recommends this value on the basis of 564 tests on columns carried out during 1927 to 1933 at Lehigh and Illinois universities (MacGregor & Wight 2005). The nominal concentric load capacity of a column (P_o) can be expressed as

$$P_o = 0.85 f'_c \left(A_g - A_{st}\right) + A_{st} f_y \qquad (1.1)$$

Rearranging (1.1), effective strength of concrete ($f_{ce}^{'}$) can be defined as below where two or more types of concretes are used in a column.

$$f_{ce}^{'} = \frac{P_o - A_{st} f_y}{0.85\left(A_g - A_{st}\right)} \qquad (1.2)$$

1.2.1 ACI Code Provisions

The ACI in its Building Code Requirements for Reinforced Concrete (ACI 318-05) deals with this problem in its Section 10.15. The requirements of this Section are based on a paper, "Effect of Floor Concrete Strength on Column Strength" by Bianchini et al (1960), which state;

10.15 – When the specified compressive strength of concrete in a column is greater than 1.4 times that specified for a floor system, transmission of load through the floor system shall be provided by 10.15.1, 10.15.2, 10.15.3.

10.15.1 – Concrete of strength specified for the column shall be placed in the floor at the column location. Top surface of the column concrete shall extend 2 ft into the slab from face of column. Column concrete shall be well integrated with floor concrete, and shall be placed in accordance with 6.4.5 and 6.4.6.

10.15.2 – Strength of a column through a floor system shall be based on the lower value of concrete strength with vertical dowels and spirals as required.

10.15.3 – For columns laterally supported on four sides by beams of approximately equal depth or by slabs, it shall be permitted to base strength of the column on an assumed concrete strength in the column joint equal to 75 percent of column concrete strength plus 35 percent of floor concrete strength. In the application of 10.15.3, the ratio of column concrete strength to slab concrete strength shall not be taken greater than 2.5 for design.

This section, unchanged since 1963, implies that if column concrete strength does not exceed the floor concrete strength by more than 40 percent, no special precautions need be taken. For higher column concrete strengths, provisions 10.15.1 or 10.15.2 should be followed for corner or edge columns. Methods in 10.15.1, 10.15.2, 10.15.3 are applicable to interior columns with adequate restraint on all four sides. The upper limit of 2.5 in article 10.15.3 was included following the research work by Ospina and Alexander (1998) which showed that heavily loaded slabs do not provide effective confinement as in lightly loaded slabs when ratios of column concrete strength to slab concrete strength exceeded 2.5.

In other words;

When $f'_{cc}/f'_{cs} \le 1.4$

$$f'_{ce} = f'_{cc} \qquad \text{(for interior, corner \& edge columns)} \qquad (1.3\text{-a})$$

When $f'_{cc}/f'_{cs} > 1.4$

$f'_{ce} = f'_{cs}$ (for corner & edge columns) (1.3-b)

$f'_{ce} = 0.75 f'_{cc} + 0.35 f'_{cs}$ (for interior columns) (1.3-c)

1.2.2 Previous Research

Current ACI 318 requirements of Section 10.15 are based on experimental work by Bianchini et al (1960). They tested forty five, 11 in-square specimens, representing portions of the corner, edge, and interior column-floor sections of a typical structure. Maximum concrete strength of about 8000 psi was used in column portion. The variables included type of specimen, column concrete strength (f'_{cc}), and floor concrete strength (f'_{cs}). The study concluded that the column strength is a function of the ratio of column concrete strength to floor concrete strength and the number of restrained edges tributary to the column; they observed no reduction in column strength for ratio of f'_{cc}/f'_{cs} up to 1.4 for all type of specimens. Although they discussed about other influencing parameters but did not include them in their research work like relative thickness of floor, size of specimen, percentage of reinforcement and load eccentricity.

The second known work on the subject matter was that by Gamble and Klinar (1991) at the University of Illinois, Urbana who tested twelve specimens representing flat plate floor systems under compressive axial load. Of the twelve slab-column specimens, six were edge columns and six represented interior columns. The study, aimed at investigating the general adequacy of Section 10.15 of ACI Code concluded that code provisions are unconservative for higher ratios of f'_{cc}/f'_{cs} for interior columns and proposed equations (1.4) and (1.5) for f'_{cc}/f'_{cs} >1.4 to calculate effective concrete strength value to be used in calculating capacity of edge and interior columns respectively. The equations are

$$f'_{ce} = 0.85 f'_{cs} + 0.32 f'_{cc} \quad (1.4)$$

$$f'_{ce} = 0.67 f'_{cs} + 0.47 f'_{cc} \quad (1.5)$$

In another experimental study at University of Illinois, six specimens with aspect ratio (h/b) of 0.7 were tested by Kayani (1991) to understand the load transfer mechanism of high strength concrete column through a layer of lower strength slab concrete and to determine the effects of confinement on behaviour of slab concrete. The specimens, having 55 in depth and constant floor thickness of 7 inches, consisted of four sandwiched type of columns and two edge columns. Kayani validated the observation by Gamble and Klinar that Section 10.15 provisions overestimate the apparent floor concrete strength in case of interior columns. He also suggested that allowable ratio of 1.4 between the two concrete strengths within which there is no requirement to reduce the column capacity is inappropriate. Treating the specimens as composite materials, he used mechanics of materials approach for developing following equation to calculate the effective concrete strength applicable to all kind of columns.

$$f'_{ce} = 2.0\lambda_G \frac{f'_{cc} f'_{cs}}{f'_{cc} + f'_{cs}} \tag{1.6}$$

Where;

$\lambda_G = 0.90, 1.00, 1.25$ for corner, edge and interior columns respectively.

In 1992 Chuan-Chien Shu and Neil M. Hawkins proposed an expression (1.7) for computing effective concrete strength of a sandwich column and concluded that current ACI provisions for f'_{cc} / f'_{cs}>1.4 are overly conservative for edge and corner columns and may not be safe for interior columns in certain cases. Their results were based on tests conducted on 54 sandwich column specimens, consisting of high strength concrete ends framing a lower strength and variable depth concrete central section. They also observed that the restraint provided by higher strength concrete column stubs to the slab portion increased its capacity. The magnitude of increase varied with the ratio of slab depth to column's least dimension.

$$f'_{ce} = f'_{cs} + A\left(f'_{cc} - f'_{cs}\right) \tag{1.7}$$

Where; $A = \dfrac{1}{\left(0.4 + 2.66\dfrac{h}{b}\right)}$

Twenty reinforced concrete interior slab-column connection specimens were tested by Ospina & Alexander (1997) as a continuation of above studies. The key feature of this testing program was application of slab loading. The study reported that effective strength of slab-column joint reduced, with application of slab loading and increase in aspect ratio. It also reported that the current design equation for effective concrete strength of ACI Code for interior columns which is based on tests conducted on unloaded slabs is unsafe for joints with high aspect ratios. Acknowledging the effect of aspect ratio like previous studies (Shu & Hawkins 1992), a new design equation was proposed for interior columns having f'_{cc} / f'_{cs} >1.4 as follows.

$$f'_{ce} = \left(\frac{0.25}{h/b}\right) f'_{cc} + \left(1.4 - \frac{0.35}{h/b}\right) f'_{cs} \tag{1.8}$$

Where; h/b ≥ 1/3 while applying above equation.

To examine the effects of parameters like placement of fibres in lower strength concrete and confinement effect from surrounding slab on axial capacity of columns, twelve column specimens were tested in compression by McHarg et al (2000). The increase in axial compressive strength and ductility of columns due to confining effect from surrounding slab was acknowledged; gain in strength and stiffness was also observed with the addition of fiber-reinforced concrete in the slab-column specimens. They observed no detrimental effect on axial load capacity of column even for ratios of f'_{cc} / f'_{cs} = 2.2, when closely spaced reinforcing bars were used combined with fibre-reinforced concrete.

In an other research project at the University of Melbourne, Australia in 2004, a total of six sandwich column specimens were tested by Lee & Mendis to investigate the effects of aspect ratio (h/b) and column rectangularity on the effective concrete strength (f'_{ce}) of high strength concrete corner columns intersected by weaker slabs. The aspect ratio varied from 0.3 to 1.14 with maximum of 12600 psi concrete strength. Investigators of the project confirmed the observation of previous work (Shu & Hawkins 1992) that axial capacity of column decreased with an increase in aspect ratio. They observed that specimens with square geometry had higher values of peak stress to slab concrete strength ratios when compared with rectangular columns and concluded that it would be inaccurate if aspect ratio is not considered in estimating the effective compressive strength of joint.

A research work, second of its kind (first one by Ospina & Alexander 1998) was carried out in year 2005 by Shah et al on six interior and one sandwich column specimens where slab was also loaded to simulate the realistic conditions. They confirmed the test results from previous studies (Kayani 1991, Shu & Hawkins 1992, Ospina & Alexander 1998), i.e. reduction in the effective strength of the joint due to slab loading, overestimation of the joint effective strength for high ratios of $f_{cc}^{'} / f_{cs}^{'}$ and h/b by ACI Code, and increase in $f_{ce}^{'}$ due to confining effect. They further reported that besides the confinement from surrounding slab, provision of stirrups in the joint region of interior columns improved the joint effective strength. Also a new design expression was proposed as follows;

$$f_{ce}^{'} = 0.35 f_{cc}^{'} + 0.384\left(\frac{\rho + 4.12}{h/b + 1.47}\right)\lambda \times f_{cs}^{'} \qquad (1.9)$$

Where;

$$\lambda = \frac{f_c^{'}(Interior)}{f_c^{'}(Sandwich)}$$

Lee et al in 2007, investigated the effect of fibre-reinforced puddle concrete on axial capacity of interior columns. Six specimens were tested for the purpose. The study reported an increase of 16% in failure load and smaller cracks in slab at different levels of loading. It further reported that ACI provisions were conservative even though the slabs were applied with loads during testing.

1.3 OBJECTIVES AND SCOPE

Main objective of this research was to investigate the effect of floor thickness made of lower strength concrete than that of columns, on column capacities. The conclusions drawn in previous studies are based on a limited number of test data, necessitating additional research work. For the purpose tests were conducted on nine (09) axially loaded sandwich column specimens with varying ratios of slab thickness to width of column section. All the other parameters except floor thickness i.e. reinforcement ratio (ρ), specimens' dimensions, floor concrete strength ($f_{cs}^{'}$), column concrete strength ($f_{cc}^{'}$) were held constant. Three groups with three specimens in each group having aspect ratios 0.67, 1 and 1.33 were tested. Specimen geometry, cover, and reinforcement details are shown in Fig. 1.1 for each group.

Chapter 2

SPECIMENS, MATERIALS AND MIX DESIGN

2.1 DESCRIPTION OF SPECIMENS

The specimens were designed keeping in view the capacity of available testing machine and to maintain the similarity in characteristics with actual structural elements. The cross sectional dimensions (6in x 6in) and height of 30 in were held constant for all specimens. Aspect ratios 0.67, 1, 1.33 were selected for testing program to cover both the flat-slab and flat plate floor systems. A total of 4 # 5 bars were used as a longitudinal reinforcement in each specimen which were welded to a ½" thick, 6 square inch steel plate at bottom for proper vertical alignment. Height of specimens mentioned earlier is exclusive of this steel plate thickness. The main reinforcement were tied together using #3 bars with maximum spacing equal to that of column's least dimension. Thus the reinforcement ratio (ρ), tie size and spacing are in compliance with the ACI-Section 10.9.1 which limits reinforcement ratio in between 1% to 8% and Section 7.10.5 respectively.

2.2 CATEGORIES OF SPECIMENS

All the nine specimens were divided into three groups A, B and C, having three specimens in each group. The three specimens in each group had slab layer of 4, 6 and 8 inches, sandwiched between two column ends made up of comparatively higher strength concrete. With 4 # 5 bars as main reinforcement and gross cross sectional area of 36 in^2, each specimen had reinforcement ratio (ρ) of 3.4%. Specimen with 4" thick slab layer had aspect ratio (h/b) of 0.67, typical to that of flat plate floor system, whereas those with 6" and 8" thick layer developed an aspect ratio of 1 and 1.33 respectively. Fig.1.1 shows rest of the features of specimens in all groups.

2.3 DESIGNATION OF TEST SPECIMENS

All nine (09) specimens were divided into three groups; A, B and C having three specimens each. Each specimen is designated with three alphabets and one numeric letter. The first two letters “SC” meaning sandwich column being common for all specimens and the third alphabet indicates group name and numeric digit denotes floor-slab thickness. Thus the specimens are designated as SCA-4, SCA-6, SCA-8, SCB-4, SCB-6, SCB-8, SCC-4, SCC-6, SCC-8.

2.4 MATERIAL PROPERTIES

2.4.1 Reinforcing Steel

The longitudinal column reinforcement used for all the specimens consisted of 4 # 5 deformed bars with #3 deformed bars as stirrups, both having yield strength of 60 ksi. The stress strain curves for these bars are shown in Fig. 2.1.

2.4.2 Column Concrete Materials

- **Cement.** Ordinary Portland Cement Type-1.
- **Coarse aggregate.** Kiryana hill aggregate with maximum particle size of ½” and specific gravity value of 2.91 was used. The gradation and sieve analysis was determined in accordance with ASTM C 136-05 which is tabulated in Table 2.1 and illustrated in Fig. 2.2
- **Fine aggregate.** Lawrencepur sand with fineness modulus of 2.5 was used as fine aggregate. Sieve analysis for fine aggregate was also performed in accordance with ASTM C 136-05, tabulated in Table 2.2 and graphically shown in Fig. 2.3.
- **Silica fume.** Silica fume, also known as microsilica, or condensed silica fume, is a highly pozzolanic material which produces more dense cement matrix resulting in high strength and durable concrete. The technical data provided by the supplier for this material is as under;

Appearance: Grey Powder

Specific gravity: 2.2-2.3

Mean particle size: 0.5 μm

- **Superplasticizer.** Incorporation of silica fume increases the water requirement in concrete appreciably unless high range water reducing admixtures or superplasticizers are used. Therefore Sika Viscorete-1 which is a third generation high performance superplasticizer based on modified carboxylic acids was used for improving workability and controlling water cement ratio at the same time. The material in its liquid form has density of approximately 1.05 kg/lit at 25^0 Celsius.
- **Water.** Potable water was used for concrete mix.

2.4.3 Floor Concrete Materials

- **Cement.** Ordinary Portland Cement Type-1.
- **Coarse aggregate.** Margalla crush with maximum particle size of ¾" was used as coarse aggregate. This aggregate had oven dry unit weight of 101.7 pcf and specific gravity value of 2.7.
- **Fine aggregate.** Lawrencepur sand with fineness modulus of 2.5 was used.
- **Water.** Potable water was used for concrete mix.

2.4.4 Antisol-E 15

Antisol–E 15 is a liquid, paraffin based curing compound for preventing water loss in concrete. This compound was used to avoid the continuous sprinkling of water regularly which is otherwise necessary to prevent water loss from specimens. Other technical data of compound supplied by the manufacturer is as under;

Type: Paraffin based,

Form: White liquid

Density at 25 ^oC: 0.96 kg/lit

2.4.5 Strain Gauges

A strain gauge is a device used to measure deformation (strain) of an object. The most common type of strain gauge consists of an insulating flexible backing which supports a metallic foil pattern. The gauge is attached to the object by a suitable adhesive. As the object is deformed, the foil is deformed, causing its electrical resistance to change. This resistance change is related to strain by the gauge factor. For this experimental work, 3 foil type strain gauges were used for every specimen. Fig. 2.4 shows the magnified view of one such strain gauge.

2.5 CONCRETE MIX DESIGN

ACI mix design procedure was followed to design the mix for floor concrete with target compressive strength of 3500 psi, where as trial mixes were prepared for producing higher strength concrete for column ends.

2.5.1 Floor Concrete Mix Design – ACI Method

Following preliminary data was collected after performing tests on fine and coarse aggregates as per ASTM specifications.

Fineness modulus of sand	2.5	ASTM C-136
Unit weight of C.A (O. Dry)	101.7 lb/cft	ASTM C-29
Specific gravity of C.A	2.7	ASTM C-127
Absorption capacities C.A	1 %	ASTM C-127
Specific gravity of fine agg:	2.65	ASTM C-128

Mix design steps;

Step 01 Slump value of 3 inches is selected from Table 2.3 for columns.

Step 02 Amount of approximate mixing water ($340\, lb/yd^3$) and value of entrapped air content (2%) against maximum aggregate size i.e. ¾” and selected slump value in step 1 are taken from Table 2.4.

Step 03 From Table 2.5, w/c ratio of 0.63 is taken for target compressive strength value of 3500psi, w/c ratio 0.63 is selected. Amount of cement is calculated as follows.

$$\Rightarrow \frac{w}{c} = 0.63$$
$$\Rightarrow c = \frac{340}{0.63} lb / yd^3 = 539.68 lb / yd^3$$

Step 04 The fineness modulus value of fine aggregate and maximum coarse aggregate size, volume of dry rodded coarse aggregate per unit volume of concrete is found from Table 2.6 as 0.65 cubic yard per cubic yard of concrete.

Thus coarse aggregate volume $= 0.65 \times 27 \frac{cft}{yd^3} = 17.55 \frac{cft}{yd^3}$

Oven dry weight of coarse aggregate $= 17.55 \frac{cft}{yd^3} \times 101.7 \frac{lb}{cft} = 1784.835 \frac{lb}{yd^3}$

SSD weight of coarse aggregate = Oven dry weight x (1+absorption capacity)

$$\Rightarrow Weight_{SSD} = 1784.835 \frac{lb}{yd^3} \times (1 + 0.01) = 1802.68 \frac{lb}{yd^3}$$

Step 05 The above calculated quantities of water, cement, air and coarse aggregate are converted to volumes and then subtracted from total concrete volume to find out the volume of fine aggregate.

$$\Rightarrow Water = 340 \frac{lb}{yd^3} \div 62.4 \frac{lb}{cft} = 5.45 \frac{cft}{yd^3}$$
$$\Rightarrow Cement = 539.68 \frac{lb}{yd^3} \div (62.4 \frac{lb}{cft} \times 3.15) = 2.75 \frac{cft}{yd^3}$$
$$\Rightarrow C.Agg = 1802.68 \frac{lb}{yd^3} \div (62.4 \frac{lb}{cft} \times 2.7) = 10.7 \frac{cft}{yd^3}$$
$$\Rightarrow Air = .002 \times 27 \frac{cft}{yd^3} = 0.54 \frac{cft}{yd^3}$$
$$\Rightarrow Total = 19.43 \frac{cft}{yd^3}$$

Therefore fine aggregate must occupy volume of 27-19.43 = 7.566 $\frac{cft}{yd^3}$

The SSD weight of fine aggregate is $7.566 \times 2.65 \times 62.4 = 1251 lb$

The adjustment of moisture in aggregate as per ACI was intentionally left since design was not to achieve the exact target strength but a mix that had strength in the range of 2.5 to 3 ksi.

2.5.2 Column Concrete Mix

After testing different trial mixes with varying silica fume contents and keeping in view minimum target strength of 6000psi at 7 days, following mix was selected for casting of test specimens.

- Cement = 47 lb/cft
- Silica Fume = 5.23 lb/cft
- Sand = 33 lb/cft
- Coarse aggregate = 50.6 lb/cft
- Water = 12 lb/cft
- Super plasticizer = 1.97 lb/cft

Chapter 3

FABRICATION AND TESTING

3.1 INSTRUMENTATION

The specimens were cast in upright position; the main reinforcement i.e. 4#5 bars were first welded to ½” thick steel plate and then tied with #3 bars with the help of steel binding wire. All specimens were instrumented with three (03) strain gauges on the main reinforcing bar; one gauge in top column, one in sandwiched portion of slab and one in bottom column area. Before mounting strain gauges, areas of the reinforcing bars were leveled with the help of grinder to ensure proper contact of gauge with reinforcement. Fig. 3.1 (a) shows the leveled surfaces of reinforcement where gauges were applied. The gauges were then bonded using an adhesive with the reinforcement. After soldering the lead wires with gauges (Fig. 3.1 (b)), they were water proofed using silicon sealant to avoid chances of damage during concreting. Fig. 3.2 shows different views of specimen after soldering.

3.2 FABRICATION OF SPECIMENS

Steel formwork was used for casting all the specimens. As mentioned earlier the steel cage was welded with a ½” thick steel plate. All concrete was mixed in a small rotating type drum mixer. Mixing of concrete for columns was done for 8 to 10 minutes to ensure proper mixing and dispersion of silica fume and high range water reducing admixture with other ingredients. Mixing of concrete for slab portion took 3 to 5 minutes. After the concrete was placed in the forms and control cylinder molds, it was vibrated for approximately 15 sec with an internal high frequency rod vibrator in case of specimens, whereas vibration table was used for compacting concrete in cylinders.

Each specimen was cast in three stages; the lower column, the slab, and finally the upper column. Slump was determined immediately after mixing (Fig. 3.3). Every stage in casting process had a time difference of at least one day. Thus the entire casting process for each group of specimens took three to five days. The first stage for casting lower column involved form work setup, pouring of concrete, compaction using mechanical vibrator, formwork removal after at least one day and finally application of curing compound. The second and third stage also involved the above same operations with one additional work of placing ½" thick steel plate in third stage immediately after pouring of concrete. Six 4 x 8-in control cylinders were also cast from each batch (Fig. 3.4). One batch was required for casting specimens at each stage and 6 control cylinders. Table 3.1 shows the dates of casting specimens.

Specimens were then carefully transported to structural laboratory of UET Peshawar for final testing ensuring no damage is caused to the specimens and gauge wires during transportation and handling process.

3.3 TESTING OF SPECIMENS

3.3.1 Test Setup

All specimens were tested in axial compression in a 200 tons capacity compression testing machine. First, the test specimens were installed in the testing machine and the strain gauge wires connected to the data logger. They were then centered and aligned with the machine axis. A typical test setup is shown in Fig. 3.5.

3.3.2 Testing Procedure

Three cylinders cast from the column concrete and slab concrete batches were tested first to obtain the axial compressive strength. This was followed by compressive testing of sandwich column specimens. The load was applied in increments of 10 tons. During the test, behavior of specimens was carefully monitored, cracks were marked on their appearance along with load readings. Strain gauge readings were also recorded after each load increment. After failure of the specimens, the crushed concrete around the failure area was removed to observe the behavior of the reinforcement.

Each test took approximately 25 minutes from the time of initiation of the load until the completion of the test.

Chapter 4

EXPERIMENTAL RESULTS

4.1 CONCRETE STRENGTHS

For each batch of concrete, six (06) control cylinders (4” x 8”) were cast; three for getting standard 28-days strength values and three were tested on final testing day. Cylinders were properly capped and centered before load application. A loading rate of 35 psi was held constant throughout their testing. Tables 4.1 and 4.2 show these test results. As can be seen from these results that cylindrical strength varied significantly in case of column concrete. The average strength of three cylinders is used in calculations.

4.2 SPECIMENS’ BEHAVIOR

Over 95 percent of all building columns in nonseismic regions are tied columns. For these columns, vertical cracks and crushing develop in the concrete shell outside the ties at maximum load and cover concrete spalls off. When this occurs, the capacity of the core that remains is less than the load on the column. The concrete core is then crushed, and the reinforcement buckles outward between ties. This occurs suddenly, without warning, in a brittle manner.

Most of the specimens in current experimental work also failed due to buckling of the longitudinal bars and crushing of slab concrete. The buckling of longitudinal bars and crushing of slab concrete took place almost simultaneously and suddenly. Initial cracks initiated in the column ends near loading plates of testing machine. Cracks appeared in slab area and progressed vertically until spalling of cover concrete.

The summary of test results for all specimens that include the aspect ratio h/b, the ratios of f'_{cc}/f'_{cs}, the maximum applied column load (P_t), the apparent concrete strength (f'_{cp}), and the ratios of f'_{cp}/f'_{cs} are tabulated in Table 4.3.

4.2.1 Category "A" Specimens

Specimens of this series failed well below the expected axial load capacity of column. Both the specimens failed in upper column portion. The possible reasons for failure at such lower loading could be the improper contact of top column steel plate with loading plate of testing machine or defective concrete casting.

- **SCA – 4**

  Loading started at 12:30 AM on 8th May, 2008 with only one strain gauge in top column portion working properly. The load was applied in increments of 11 kips till the load reached to its maximum value of 99 kips. Cracks in the specimen were first observed in the top column portion near steel plate (Fig. 4.1-b) at the load of 88 kips. Spalling of cover concrete in the same area (top column) started at the load of 99 kips. The specimen finally failed by crushing of top column concrete at 101 kips (Fig. 4.1-d).

- **SCA – 6**

  It was tested at 10:50 AM with all the three strain gauges in working condition. The load was applied as in previous case with maximum load reaching to 136 kips. Crack formation started in top column immediately below the steel plate at the load of 88 kips (Fig. 4.2-b). Spalling of cover concrete was observed at 121 kips and the final failure with crushing of top column concrete at the load of 136 kips.

- **SCA – 8**

  The initial load application started at 12:50 PM, with only one gauge in slab portion working properly. With same loading conditions, cracking in the top column portion was observed at the load of 68 kips (Fig. 4.3-a). The specimen behaved similarly to two previous tests except that load dropped from 99 kips to 88 kips at failure.

4.2.2 Category "B" Specimens

Specimens of this group failed comparatively at higher loading than category "A" specimen. Specimens SCB -4 and SCB – 6 failed in slab portions with simultaneous buckling of main reinforcement and crushing of concrete, where as SCB – 8 failed only by crushing of top column concrete.

- **SCB – 4**

Loading of the specimen started at 1:33 PM, with only one strain gauge in bottom column portion working properly. Load was applied in increments of 11 kips. Cracking of the specimen started in slab area at the load of 158 kips, which later on also appeared in top column area at 168 kips (Fig. 4.4-b). Specimen finally failed by buckling of bars and crushing of slab concrete (Fig. 4.4-c) at 132 kips.

- **SCB – 6**

Loading started at 02:07 PM, with top and bottom column strain gauges in working condition. Cracking started in both the column ends simultaneously first at 187.4 kips. At 189.6 kips, a wedge shape type of cover concrete in top column portion started detaching from specimen (Fig. 4.5-b), but the final failure of specimen was observed in slab portion at 190 kips with concrete crushing and buckling of main reinforcement (Fig. 4.5-c, d)

- **SCB – 8**

The initial load application started at 01:50 with only one strain gauge working properly in bottom column portion. Cracks started appearing near free end of top column portion at the load of 110 kips. As these cracks started propagating vertically downward, spalling of cover concrete was observed at 139 kips. Load at this stage dropped but later on stabilized to its new high value of 141 kips. Specimen failed in upper column portion with crushing of concrete at this load.

4.2.3 Category "C" Specimens

All the specimen of this group failed in slab region. With same mix design, specimens of this group could not achieve the desired concrete strength as was expected in previous groups.

- **SCC – 4**

The initial load application started at 01:10 PM. All the three gauges were working properly. Cracks started appearing first in slab region at the load of 132 kips (Fig. 4.7-b). Cover concrete just above the sandwiched portion spalled off at the load of 165 kips. At this stage load dropped but then stabilized and achieved its maximum value of 168.8 kips where the specimen finally failed in slab region with both buckling of reinforcement and crushing of concrete occurring at the same time (Fig. 4.7-d).

- **SCC – 6**

Loading for the specimen started at 02:26. Only one strain gauge in bottom column portion was working. It failed like SCC – 4, but at lower load. Cracks started appearing in slab region at the load of 121 kips. Specimen failed at the ultimate load of 163 kips after spalling of cover concrete at 160 kips.

- **SCC – 8**

The initial load application started at 02: 45 PM. For the specimen only two strain gauges in slab and bottom column portion were working. Specimen failed comparatively at lower load as compared with other specimens of this category. Formation of vertical cracks started appearing in slab region at the load of 121 kips. Cover concrete spalled off at 158 kips. The specimen finally failed like its group members with buckling of bars and crushing of concrete; Fig. 4.9 shows different loading stages and the final failure of this specimen

4.3 STRAIN MEASUREMENTS

Strains were measured using foil type strain gauges as shown in Fig. 2.4. Total 27 strain gauges were installed, out of which only 15 worked properly. Strain values obtained are tabulated separately for each specimen in the Tables 4.4 – 4.12. These strain values for every gauge are also plotted against loads in Fig. 4.10 – 4.18.

Chapter 5

ANALYSIS OF RESULTS

5.1 GENERAL

Most of the specimens confirmed the established behavior of axially loaded columns. Specimens with higher aspect ratios (*h/b*), failed at lower loads. The expected reasons for unusual behavior of specimens of SCA series have already been discussed in section 4.2.1. Additional data, widening the ranges for floor thickness (h), and ratio of column to floor concrete strengths (f'_{cc}/f'_{cs}) was obtained in the experimental program. The apparent concrete strength has been calculated as under;

$$f'_{cp} = \frac{P_t - A_{st} f_y}{0.85\left(A_g - A_{st}\right)}$$

The current experimental results combined with the previous available data have been used for finding out the suitable expression for effective concrete strength, f'_{ce}.

5.2 COMPOSITE MATERIAL ANALOGY

The proposed equation (1.6) by Kayani (1991) is based on the principles of composite materials. The same approach has been refined to determine its effectiveness and applicability. The basic principle or analogy is that the column with two different grades of concrete should behave like composite material made of two kinds of materials. Composites are generally used because they have desirable properties which can not be achieved by either of the constituent materials acting alone. The most common example is the fibrous composite consisting of reinforcing fibers embedded in a binder, or matrix material. Composites have generally long fibers in the direction of load since the primary function of the composites is to utilize the extra ordinary strength and stiffness properties of the fibers usually along their major axes. Fibers alone cannot support longitudinal compressive loads and their transverse mechanical properties are generally not as good as the corresponding longitudinal properties. Thus, fibers are generally useless as structural materials unless they are held together in a structural unit with a binder or matrix material. A matrix is usually a weak material whose primary function is to protect and support the fibers and to provide a means of distributing among and transmitting load between the fibers (Gibson 1994). Fig. 5.1 shows the representative volume element and simple states of stress in elementary mechanics of materials models. A representative volume element (RVE) is the generic composite block consisting of fiber material bonded to matrix material.

A composite material is obviously heterogeneous at the constituent material level, with constitutive relationships changing from point to point. For a representative volume element in a heterogeneous composite, volume averaged stresses can be related to the volume averaged strains by the effective moduli of an equivalent homogenous material. Fig. 5.2 shows comparison between RVE and a sandwich column and also equivalent homogenous material with effective transverse modulus of elasticity (E_y).

When the representative volume element (RVE) or sandwich column is subjected to transverse normal stress (σ_y) as shown in Fig. 5.2, the response is governed by the effective transverse modulus, E_y. Geometric compatibility requires that the total transverse composite displacement, δ_{cy}, must equal the sum of the corresponding transverse displacements in the fiber, δ_{fy}, and the matrix, δ_{my} :

=> $$\delta_{cy} = \delta_{fy} + \delta_{my}$$

=> $$\varepsilon_{cy} L = \varepsilon_{fy} L_f + \varepsilon_{my} L_m \qquad (\because \varepsilon = \delta/L)$$

=> $$\varepsilon_{cy} = \varepsilon_{fy} \times \frac{L_f}{L} + \varepsilon_{my} \times \frac{L_m}{L}$$

Using definition of Hook's Law;

=> $$\frac{f_y^{'}}{E_y} = \frac{f_{fy}^{'}}{E_{fy}} \times \frac{L_f}{L} + \frac{f_{my}^{'}}{E_{my}} \times \frac{L_{my}}{L}$$

If we assume that the stresses in the composite, matrix and fiber are all equal then above equation reduces to;

=> $$\frac{1}{E_y} = \frac{L_f}{L \times E_{fy}} + \frac{L_{my}}{L \times E_{my}} \qquad (5.1\text{-a})$$

From the Fig. 5.1(a), it would seem that the assumption of equal stresses is valid because equilibrium requires that the forces must be equal for the series arrangement and both fiber and matrix block have equal areas normal to the Y-direction.

Using sandwich column's notations, equation 5.1-a becomes;

=> $$\frac{1}{E_y} = \frac{h}{L \times E_s} + \frac{L_{cc}}{L \times E_{cc}}$$

=> $$\frac{1}{E_y} = \frac{h}{L \times E_s} + \frac{(L-h)}{L \times E_{cc}} \qquad (\because L = h + L_{cc})$$

=> $$\frac{1}{E_y} = \frac{hE_{cc} + E_s L - E_s h}{LE_s E_{cc}}$$

=> $$E_y = \frac{LE_s E_{cc}}{hE_{cc} + E_s L - E_s h}$$

=> $$E_y = \frac{LE_s E_{cc}}{h(E_{cc} - E_s) + E_s L}$$

Since $E_c \infty \sqrt{f_c^{'}}$ or $f_c^{'}$

=> $$\sqrt{f_{ce}^{'}} = \frac{L\sqrt{f_{cc}^{'} f_{cs}^{'}}}{h(\sqrt{f_{cc}^{'}} - \sqrt{f_{cs}^{'}}) + \sqrt{f_{cs}^{'}} \times L}$$

=> Or $$f_{ce}^{'} = \frac{Lf_{cc}^{'} f_{cs}^{'}}{h(f_{cc}^{'} - f_{cs}^{'}) + f_{cs}^{'} \times L} \qquad (5.1\text{-b})$$

It is assumed that length of column segment in the composite can not be more than least dimension of column cross section. Therefore equation 5.1-b is modified as

=> $$f'_{ce} = \frac{bf'_{cc} f'_{cs}}{h(f'_{cc} - f'_{cs}) + f'_{cs} b}$$

=> $$f'_{ce} = \frac{f'_{cc} f'_{cs}}{\frac{h}{b}(f'_{cc} - f'_{cs}) + f'_{cs}} \qquad (5.2)$$

5.3 ANALYSIS OF DATA

5.3.1 Corner Columns

According to Kayani (1991), Lee & Mendis (2004), sandwich columns adequately represent corner column slab joints. Therefore the previous data of sandwich and corner columns by different researchers (Bianchini et al (1960), Kayani (1991), Shu & Hawkins (1992), McHarg et al (2000), Shah et al (2005), Lee et al (2007)) have been used for analysis purposes. For corner columns ACI deals the issue of using different grades of concrete in a system in its Section 10.15.1 and 10.15.2, which recommends use of puddle concrete and lower value of concrete strength, if the ratio of (f'_{cc}/f'_{cs}) exceeds 1.4. And if this ratio (f'_{cc}/f'_{cs}) is less than or equal to 1.4, no reduction in column concrete strength is expected. In other words;

1. When $f'_{cc}/f'_{cs} \leq 1.4$

   $f'_{ce} = f'_{cc}$

2. When $f'_{cc}/f'_{cs} > 1.4$

   $f'_{ce} = f'_{cs}$

To compare the equation 5.2 with the design expressions in equations 1.6 by Kayani (1991), 1.7 by Shu & Hawkins (1992) and 1.3 of of ACI 318-05; the current test data and previous data are used. Table 5.1 shows the comparison.

h/b in equation 5.2 has to be less than 1, because it is believed that poisson effects of two concretes affect each other in that range. Fig 5.3-5.6, show apparent concrete strength of samples plotted against the calculated values from proposed expression (equation 5.2). The theoretical line at 45^{o} is also drawn on these figures along with the data points plotted.

Experimental results of all SCA series specimens and SCB-8 have not been included in data analysis because of unacceptable test results. The failure loads from tests on these specimens are substantially less than the column capacity calculated with floor concrete strength which is not possible. From Table 5.1 with overall test to predicted ratio of 1.44, the ACI Code is found to be too conservative, particularly for ratios of ($f_{cc}^{'} / f_{cs}^{'}$) greater than 1.5 (Fig. 5.7). Proposed equation by Kayani presents much better results as compared to the ACI and also has less value of standard deviation (standard deviation is a measure of how widely data is dispersed from the average value) than ACI.

The effective strength calculated ($f_{ce}^{'}$) using the newly proposed equation 5.2 has shown good correlation with the apparent strength of tested samples ($f_{cp}^{'}$). With an average test to predicted ration of 1.22 and standard deviation of 0.20, the proposed design equation appears to be much safer than the existing ones.

5.3.2 Edge Columns

Because of the restraint provided from three sides edge columns behave differently from corner columns. Therefore the proposed design equation (5.2) for corner columns is modified to equation (5.3) for estimating effective concrete strength of edge columns. Available test data (Bianchini et al 1960, Gamble & Klinar 1990, Kayni 1991) of edge columns fits well with the new equation. Fig. 5.8-5.12 show the plot of apparent concrete strength plotted against effective concrete strength calculated using equations 1.3, 1.4, 1.6, 1.7 and 5.3.

$$f'_{ce} = \frac{f_{cc}^{'} f_{cs}^{'}}{0.7\dfrac{h}{b}(f_{cc}^{'} - f_{cs}^{'}) + f_{cs}^{'}} \qquad \text{for } h/b \le 1 \qquad (5.3)$$

5.3.3 Interior Columns

Interior columns behave differently from corner or edge columns because of restraint provided by the diaphragm from all four directions. The floor concrete is expected to gain some strength due to the confinement provided by the slab. The tested interior column specimens by MacHarg et al (2000) had effective concrete strengths about 29-43% greater than the effective strengths of the isolated column specimens. Therefore equations 5.2 and 5.3 should not be used for estimating interior columns capacity.

From available test data (Bianchini et al 1960, Gamble & Klinar 1990, Ospina & Alexander 1998, Shah et al (2005), Lee et al (2007), McHarg et al (2000)), equation 5.2 is modified for interior columns as:

$$f'_{ce} = \frac{f'_{cc} f'_{cs}}{0.35\frac{h}{b}(f'_{cc} - f'_{cs}) + f'_{cs}} \qquad \text{for } h/b \le 1 \qquad (5.4)$$

The proposed equation is applicable to the test data available. ACI 318-05 expression overestimates the capacity of interior columns as shown in Fig. 5.13. The proposed equation (1.6) by Kayani (1991) results in higher mean and standard deviation values than equation (5.4) whereas expression (1.4) proposed by Gamble & Klinar overestimates the column capacity in some cases as shown in Fig. 5.14.

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

Overall nine (09) specimens were tested during the present investigation. These specimens were divided into three groups, with three specimens in each group. To see the effect of aspect ratio (h/b) on axial behavior of columns, every specimen in each group was cast with different floor thickness, made of lower concrete strength than the column ends. The specimens performed as per established behavior of axially loaded columns i.e. they failed due to buckling of the longitudinal bars and crushing of slab concrete, except those discarded due to technical reasons. The test results have been presented in Table 5.1 and data has been analyzed. The test results of this study, together with the previously conducted tests on the subject were analyzed to investigate the transfer of load from higher strength concrete columns through floors of relatively lower strength concrete.

6.2 CONCLUSIONS

The analysis and discussion of test results lead to the following conclusions:

1. Specimens of SCB and SCC series confirm that the effective strength of an axially loaded column intervened by lower strength concrete floor is influenced by its aspect ratio, h/b. As the aspect ratio increases, the effective strength of the joint decreases.

2. The ACI 318-05, Section 10.15, provisions for $f'_{cc}/f'_{cs} > 1.4$ are overly conservative for corner and edge columns. However, it overestimates capacity of interior columns when difference between tow concrete strengths is substantial.

3. Mechanics of composite materials should be used to predict the response of slab-column joints to axial loads.

4. The expressions in equations 5.2, 5.3, and 5.4 can be used for predicting the effective concrete strength of axially loaded columns. The expressions are shown to be more reliable and safe than the existing design equations.

5. ACI 318-05 should be changed for calculation of load carrying capacity of axially loaded columns as under;

$$P_o = 0.85 f'_{ce}\left(A_g - A_{st}\right) + A_{st} f_y \qquad \text{for } h/b \le 1$$

Where

$$f'_{ce} = \frac{f'_{cc} f'_{cs}}{\frac{h}{b}(f'_{cc} - f'_{cs}) + f'_{cs}} \qquad \text{(for corner columns)}$$

$$f'_{ce} = \frac{f'_{cc} f'_{cs}}{0.70\frac{h}{b}(f'_{cc} - f'_{cs}) + f'_{cs}} \qquad \text{(for edge columns)}$$

$$f'_{ce} = \frac{f'_{cc} f'_{cs}}{0.35\frac{h}{b}(f'_{cc} - f'_{cs}) + f'_{cs}} \qquad \text{(for interior columns)}$$

6.3 RECOMMENDATIONS

- Behavior of specimens in presence of slab loading for corner and edge columns in addition to the axial loads should be studied.
- Eccentric loads for investigating the behavior of sandwich column joint in presence of bending moments should be carried out.
- Additional data is required for validating/refining the proposed equation, particularly with specimens having $f'_{cc}/f'_{cs} > 3$.
- Size of the specimens should be increased to represent the physical structures.
- Confinement of the joint region (floor concrete area) with longitudinal and lateral reinforcement should be studied for improvement in behavior of floor concrete.